APPENDIX 9.B — SAG CULVERTS

9.B.1 INVERTED SIPHON

Inverted siphons (sometimes called sag culverts or sag lines) are used to convey water by gravity under roads, railroads, other structures, various types of drainage channels and depressions. An inverted siphon is a closed conduit designed to run full and under pressure. The structure should operate without excess head when flowing at design capacity.

9.B.1.1 Application

Economics and other considerations determine the feasibility of using an inverted siphon or another type of structure. The use of an elevated flume would be an alternative to an inverted siphon crossing such features as a deep roadway cut or another channel. The use of a raised grade line and culvert may be a more economical alternative to employing a siphon under a road.

9.B.1.2 Advantages and Disadvantages

Inverted siphons are easily designed, constructed and have proven to be a reliable means of water conveyance. Normally, canal erosion at the ends of the siphon is inconsequential in earth waterways provided that the transition and any erosion protection is properly designed and constructed.

Costs of design, construction and maintenance for an inverted siphon are higher than for a culvert that might be used for the same purpose. However, the cost of raising the roadway grade line may offset this higher cost.

An inverted siphon may present a hazard to life, especially in areas of high-population density. Inverted siphons cannot be used for drainage or irrigation where freezing may block the siphon’s waterway.

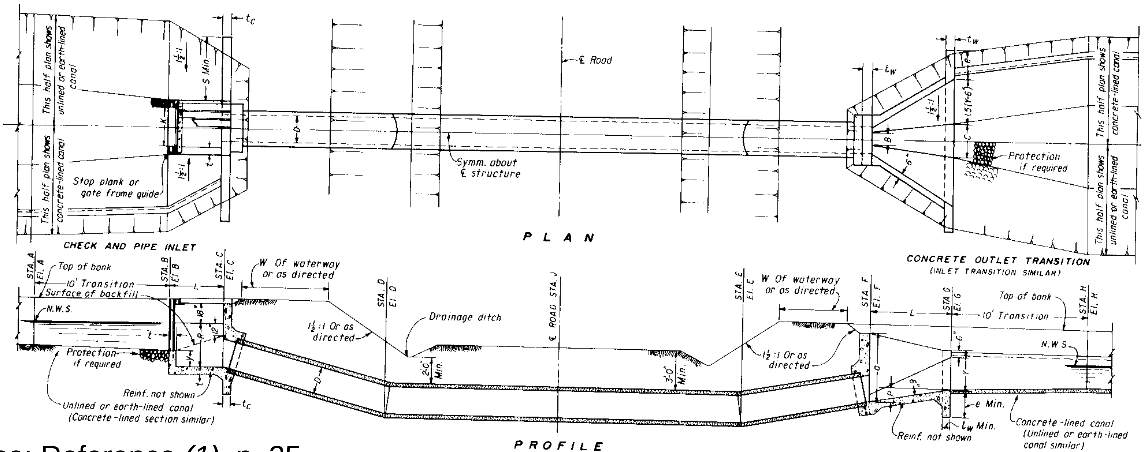
9.B.1.3 Components

9.B.1.3.1 Siphon

That portion lying between the inlet and outlet (Points C and F in Figure 9.B-1). Siphons that are subjected to internal pressure should have watertight joints. Welded smooth steel pipe with internal ceramic coating and precast reinforced concrete pressure pipe, asbestos-cement pressure pipe or reinforced plastic mortar pressure pipe are commonly used. Jointed pipe requires gaskets to ensure water tightness. Inverted siphons must be able to withstand the internal hydrostatic head measured to the centerline of the siphon.

9.B.1.3.2 Transitions

Transitions are the inlet and outlet portion of an inverted siphon (Points B to C and F to G in Figure 9.B-1). Transitions are nearly always used at the inlet and outlet of a siphon to reduce head losses and prevent canal erosion in unlined canals. Concrete transitions are preferred for this purpose, although earth has been used.



Source: Reference *(1)*, p. 25.

FIGURE 9.B-1 — Profile

9.B.1.3.3 Collars

Collars are placed at intervals along the siphon to reduce the velocity of any water moving along the outside of the siphon or through the surrounding earth thereby preventing removal of soil particles (piping) at the point of emergence. Collars are also used to discourage rodents from burrowing along the siphon.

9.B.1.3.4 Blowoff Structures

Blowoff structures are provided at or near the low point of inverted siphons to permit draining the siphon for inspection and maintenance or wintertime shutdown. Essentially, the blowoff structure consists of a valved steel drain pipe tapped into the siphon. Although less convenient and requiring higher long-term maintenance costs, short siphons may be dewatered when necessary by pumping from either end of the siphon instead of using a blowoff structure.

9.B.1.3.5 Canal Freeboard

Upstream canal freeboard is commonly provided to accommodate intercepted storm runoff, improper operation or drift blockage.

9.B.1.3.6 Wasteways

Wasteways are often placed upstream from a siphon transition to divert the canal flow in case of an emergency. Wasteways may also be integrated into the upstream transition geometry.

9.B.1.3.7 Safety Devices

Safety measures (e.g., fences, grates) must be taken near siphons to protect persons and animals from injury and loss of life. A hazard can occur both when the siphon is operational or dry.

9.B.2 SIPHON CRITERIA

The siphon profile is determined in such a way to satisfy certain requirements of cover, siphon slopes, bend angles and submergence of inlet and outlet. Following is a list of siphon cover requirements:

1. At all siphons crossing under roads, the minimum cover should be based on the structural requirements of the siphon material.

2. At siphons crossing under natural drainage channels, a minimum of 3 ft of compacted earth cover shall be provided unless studies indicate more cover is required because of projected future degradation of the channel.

3. At siphons crossing under an earth canal, a minimum of 1.5 ft of compacted earth cover shall be provided.

4. At siphons crossing under a lined canal, a minimum of 1 ft of compacted earth cover should be provided between the bottom of the canal lining and the top of the siphon.

Siphon slopes should not be steeper than 1V:2H and should not be flatter than a slope of 0.005 ft/ft.

With concrete siphons, changes in siphon grade and alignment may be made with precast elbows. Miter cutting and welding shall be used with metal siphons. Thrust blocks shall be provided at locations with poor foundation material.

9.B.2.1 Transition Criteria

The following inverted siphons shall have either a concrete inlet transition or some type of concrete inlet control structure and a concrete outlet transition:

1. All siphons crossing railroads and paved State highways.

2. All 36-in diameter and larger siphons crossing narrow (< 30 ft), unpaved, off-system roads.

3. All siphons in unlined canals with water velocities in excess of 3.5 ft/s in the siphon.

Using a standard concrete transition reduces costs by having a single transition geometry serve a range of canal and structure conditions. Because the base width and invert of standard transitions seldom match those of the canal, additional transitioning shall be accomplished when necessary with an earth transition where earth canals are involved and with a concrete lined transition where concrete-lined canals are involved. Before using a standard transition geometry, it is necessary to check and ensure that it does not cause unexpected backwater or unacceptable energy losses; when this occurs, the standard transition geometry shall be modified.

If there is a need for controlling the water surface elevation upstream from the siphon, a combined check and transition inlet, or a combined control and transition inlet, are more economical than separate structures.

9.B.2.2 Collar Criteria

Siphon collars are not to be used unless piping computations or observations of burrowing animals indicate they are needed.

9.B.2.3 Blowoff Structure Criteria

All inverted siphons > 18 in in diameter shall have a blowoff structure. A manhole or similar access is included with a blowoff on long siphons 36 in and larger in diameter to provide an intermediate access point for inspection and maintenance. To facilitate removal of any accumulated sediments and to expedite the draining process, a 8-in minimum gate valve shall be used. The drain pipe must be outfalled where drain water will not cause any damage.

9.B.2.4 Canal Freeboard Criteria

The canal bank freeboard upstream from siphons shall be increased 50% or 1.0 ft maximum to prevent washouts at these locations due to more storm runoff being taken into the canal than anticipated, by improper operation of the canal or by partial debris blockage. The increased freeboard shall extend upstream a distance from the structure such that damage caused by overtopping the canal banks would be minimal; but, in any event, the freeboard shall extend upstream from the transition inlet a minimum distance as determined by dividing the freeboard height by the canal slope, and downstream from the transition outlet a minimum distance of 50 ft or to the highway right-of-way, whichever is less.

9.B.2.5 Erosion Protection Criteria

Erosion protection shall be provided with siphons in earth canals where the critical tractive shear of the canal bed or bank is exceeded.

9.B.2.6 Wasteway Criteria

A wasteway, either separate or integral with the inlet transition, shall be provided where significant damage would occur due to escaping canal waters. Escaping waters should be conveyed to a point and released in a manner to avoid roadway or property damage.

9.B.2.7 Safety Criteria

Inlet and outlet transitions shall have hydraulically efficient, removable grates to minimize the hazard associated with human or animal egress or debris blockage both when operational as well as during non-operating periods. An alternative to grates is to fence the inlet and outlet provided that unwanted egress can be precluded. Grates shall be the preferred practice.

9.B.2.8 Design Procedure

Available head, economy and allowable siphon velocities determine the size of the siphon. To use Bernoulli's equation, it is necessary to assume dimensions for the siphon and compute the head losses (e.g., those associated with the entrance, siphon friction, siphon bends and exit). The sum of all the computed losses should be less than the available head.

9.B.2.9 Design Criteria

In general, siphon velocities should range from 3.5 to 10 ft/s, depending on available head and economic considerations and whether the siphon is considered short < 200 ft. The following velocity criteria are to be used in determining the minimum diameter of the siphon:

1. 3.5 ft/s or less for a short siphon not located under a highway with only earth transitions provided at entrance and exit,

2. 5 ft/s or less for a short siphon located under a highway with either a concrete transition or control structure provided at the inlet and a concrete transition provided at the outlet, and

3. 10 ft/s or less for a long > 200-ft siphon with either a concrete transition or control structure provided at the inlet and a concrete transition provided at the outlet.

Head losses which should be considered are as follows:

1. convergence loss in the inlet transition,

2. losses for a check structure where a check is installed in the inlet,

3. control structure losses where a control is installed in the inlet,

4. friction and bend losses in the siphon,

5. divergence loss in the outlet transition,

6. transition friction only in special or very long transitions, and

7. convergence and divergence head losses in earth transitions where required between an unlined canal and concrete transition are usually small and are thus ignored.

The total computed head loss shall be increased by (10%) as a safety factor to ensure against the possibility of the siphon causing unexpected backwater in the canal upstream from the siphon due to unforeseen operational problems.

The hydraulic head loss in a transition shall be the difference of the velocity heads in the canal and the normal to the canal centerline section of the siphon. Coefficients of velocity head for determining head losses in a broken-back type of transition shall be 0.4 for the inlet and 0.7 for the outlet so that the losses as a function of the difference in velocity heads would be $0.4\Delta h_v$ for the inlet and $0.7\Delta h_v$ for the outlet transitions.

Coefficients of velocity head for determining head losses in earth transitions from the canal to a pipe are 0.5 for the inlet and 1.0 for the outlet. Again, the losses would be a function of the difference in velocity heads or $0.5\Delta h_v$ for the inlet and $1.0\Delta h_v$ for the outlet transitions.

For minimum hydraulic loss, provide a seal of $1.5\Delta h_v$ with 3 in minimum at the siphon inlet and no submergence at the siphon outlet. The seal is equal in height to the vertical drop from the normal canal water surface to the top of the siphon opening. If the outlet seal is greater than one-sixth the height of opening at the outlet, the head loss shall be computed on the basis of a sudden enlargement and the loss for both earth and concrete outlet transitions will be $1.0\Delta h_v$.

If the siphon has both upstream and downstream concrete transitions, construct the downstream transition the same as the upstream transition whenever possible for economy.

Check or control structure losses should be considered where such facilities are provided at the inlet or outlet. These losses vary depending on the geometry and thus are outside the scope of this *Manual*; see “Design of Small Canal Structures.”

Siphon friction losses are determined by using Manning's formula.

Siphon bend losses are determined as a function of the velocity head, deflection angle, siphon diameter and radius of bend curvature.

Blowback (the discharge of trapped air) may occur under two conditions:

1. free flow inlet, or
2. with long > 200-ft siphons.

Special hydraulic considerations should be given to free-flow siphon inlets. These are inlets where, under certain conditions, the inlet will not become sealed. Such conditions may result when the canal is being operated at partial flows (flows less than design flow) or at full design flow when the actual coefficient of friction is less than assumed in design. Under such conditions, a free-flow entrance occurs and a hydraulic jump occurs in the siphon, which may cause blowback and very unsatisfactory operating conditions. Siphon slopes or diameter should be changed where blowback is expected to occur from a free-flow inlet.

On long siphons, entrained air may become trapped inside the siphon. To minimize the possibility of blowback in long siphons, place air vents at locations where air might accumulate. This procedure is ordinarily used only as a remedial measure for an existing siphon with blowback problems due to entrained air being trapped inside the siphon.

9.B.3 DESIGN STEPS

Step 1 Determine what inlet and outlet structures are required and the type and approximate size of the siphon.

Step 2 Select preliminary transition geometry.

Step 3 Make a preliminary layout of the siphon profile (Preliminary Siphon Profile) to include the siphon, required inlet and outlet structures, existing ground line, roadway geometry, the canal properties and the canal stations and elevations at the siphon ends. This layout should provide the required cover, slope and bend angles and provide siphon submergence requirements at transitions, check and siphon inlets or control and siphon inlets.

Step 4 Compute the siphon head losses using this preliminary layout and trial inlet and outlet geometry. If the head losses as computed are with the available head, it may be necessary to make some adjustment, such as siphon size or even the canal profile, provided that this is acceptable to the canal owner.

If the computed losses are greater than the difference in upstream and downstream canal water surface, the siphon will probably cause backwater in the canal upstream from the siphon. If backwater exists, the siphon size should be increased or, where acceptable to the canal owner, the canal profile revised to provide adequate head.

Step 5 If the computed losses are appreciably less than the difference in upstream and downstream canal water surface, it may be possible to decrease the size of siphon or, again if acceptable, the canal profile may be revised so that the available head is approximately the same as the head losses.

Step 6 On long > 200-ft siphons or where the inlet may not be sealed at low flows, there is the possibility of blowback and related unsatisfactory operating conditions. The inlet shall be routinely checked for any expected low flows for proper performance and adjustments made if necessary. On long siphons, vents should be considered at points where air may be trapped.

Step 7 Determine the need for such siphon components as erosion protection and siphon collars, and determine the required safety needs: wasteways, grates and fencing.

Step 8 Enter all computed siphon dimensions and angles on the Final Siphon Layout.

Determine the final transition geometry and compute actual head losses. If the actual head loss exceeds the available head, return to Step 3.

9.B.4 EXAMPLE

This Example is for irrigation flows. However, by recognizing the variance in discharges for drainage, the procedure can be used for that purpose as well. Given that an earth canal crosses a highway, an inverted siphon is the most economical type of structure for conveying water past the highway. Using the drainage survey and the proposed road geometry, first prepare a Preliminary Siphon Layout.

9.B.4.1 Given

Refer to the Preliminary Siphon Layout in Figure 9.B-2:

1. Type of waterway = earth canal

2. Feature being crossed = State highway at right angles with canal centerline

3. $Q = 15\ ft^3/s$

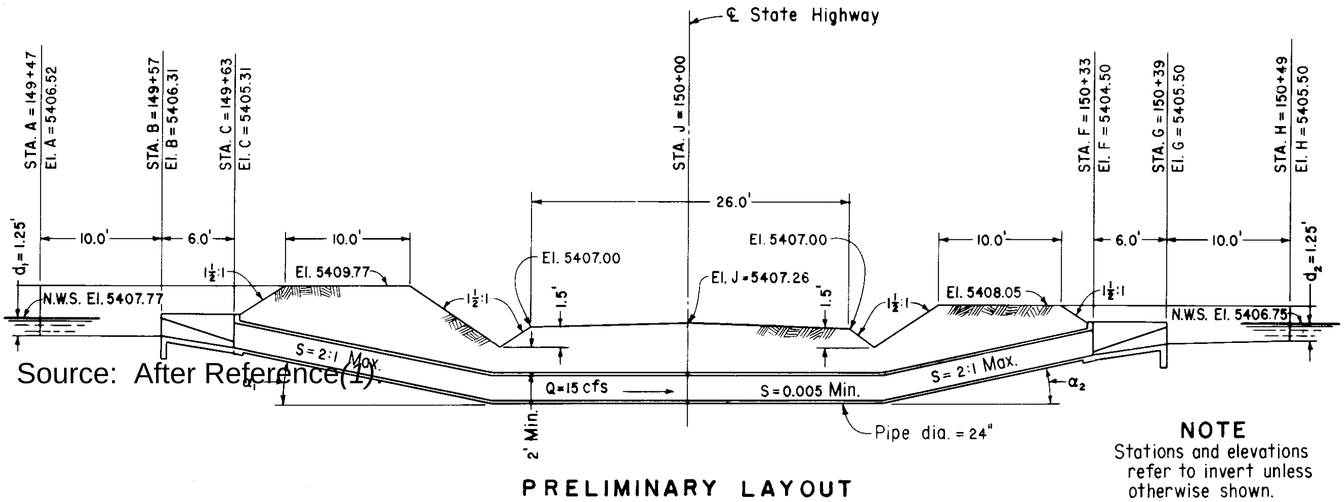
4. On canal profile, Sta. A = 149+47, Canal Invert El. A = 5406.52 ft (from drainage survey showing the canal profile)

5. $d_1 = 1.25$ ft (d_n, normal depth in canal determined using Manning's equation)
   $V_1 = 2.1$ ft/s, h = 0.07 ft

6. Normal Water Surface (NWS) El. at Sta. A = El. A + d_1 = 5406.52 + 1.25 = 5407.77 ft

7. On canal profile, Sta. M = 150+49 and Canal Invert El. M = 5405.50 ft (from the canal profile)

8. $d_2 = 1.25$ ft (d_n, normal depth in canal)
   $V_2 = 2.1$ ft/s, h = 0.07 ft



Source: After Reference (1).

FIGURE 9.B-2 — Preliminary Layout of Inverted Siphon

9. NWS El. at Sta. M = El. M + d_2 = 5405.50 + 1.25 = 5406.75 ft

10. Width of roadway = 26 ft

11. Side slopes of roadway ditch and canal embankment = 1V:1.5H

12. El. top of roadway = El. J = 5407.26 ft

13. El. edge of roadway shoulders =5407.00 ft

14. Control structure at inlet not required for turnout delivery

15. 1.5-ft deep roadway ditches

16. On canal profile at Sta. J, centerline roadway = canal St. 150+00

17. Canal bank width = 10.0 ft

18. Existing canal bank freeboard at outlet = normal canal bank freeboard = 1.3 ft

9.B.4.2 Trial Inlet-Outlet Geometry

Inlet and Outlet Structure Requirements. The criteria indicate that some kind of concrete inlet and outlet structures are required. Because a control structure is not needed at the inlet, use a concrete transition for both the inlet and the outlet. Try using the standard geometry for the inlet and outlet in Figure 9.B-3.

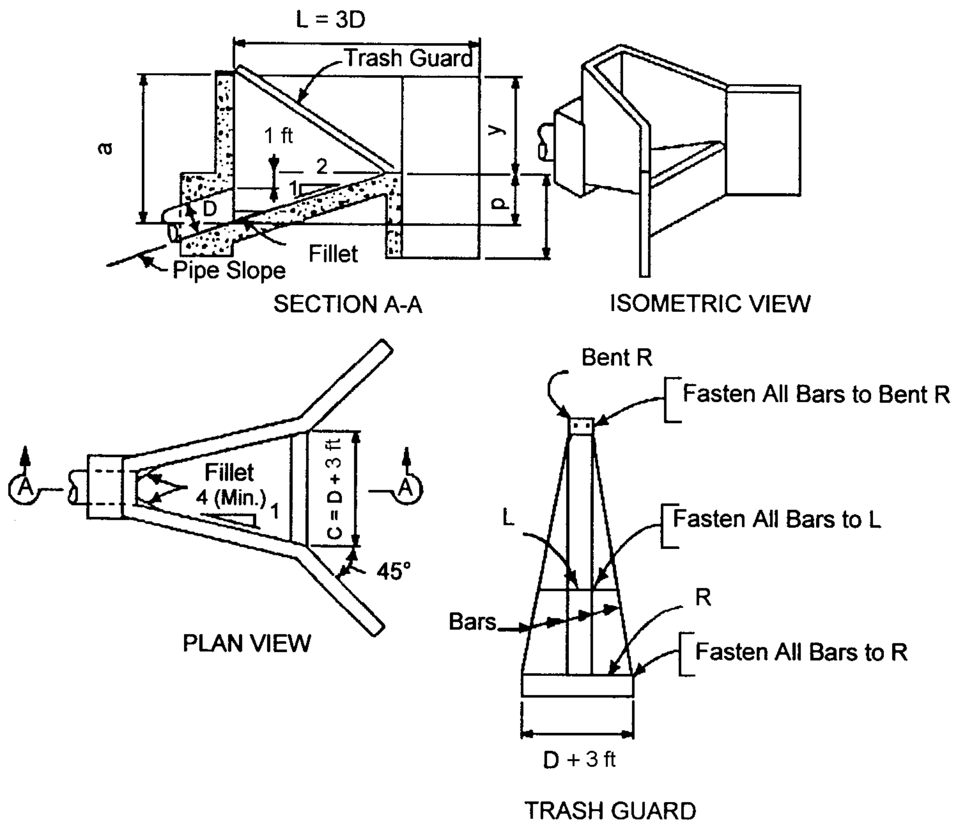


FIGURE 9.B-3 — Geometry For Inlet and Outlet

9.B.4.3 <u>Siphon Type/Trial Size</u>

This siphon will have internal pressure and will be passing under a highway; therefore, it should be either ceramic-coated (interior), welded steel pipe or precast reinforced concrete pressure pipe, asbestos-cement pressure pipe or reinforced plastic mortar pressure pipe with each of the latter having rubber gasket joints. In this Example, assume that because of availability and quality it is advantageous to use ceramic-coated welded steel pipe.

Pipe Size. For a relatively short siphon having concrete inlet and outlet transitions, the siphon using the following table would be sized for velocity of about 5 ft/s. For a discharge of 15 ft^3/s, Table 9.B-1 suggests that a 24-in diameter siphon may be used.

9.B.4.4 <u>Hydraulic Siphon Properties</u>

Hydraulic properties of a 24-in diameter siphon for Q of 15 ft^3/s:

A = area of siphon = $(0.785)(\text{dia.})^2 = 3.14\ ft^2$
V = velocity in siphon = $Q/A = 15/3.14 = 4.78$ ft/s < 5 ft/s
h = velocity head in siphon = $V^2/2g = 4.77^2/((2)(32.2)) = 0.35$ ft

where: g = gravitational acceleration

WP = wetted perimeter = $\pi D = (3.14)(2) = 6.28$ ft

R = hydraulic radius = A/WP = 3.14/6.28 = 0.5 ft

TABLE 9.B-1 — Pipe Diameter Selection Data

PIPE DIAMETER SELECTION DATA							
Max. V = 3.5 fps (Earth Transition)		Max. V = 5.0 fps (Conc. Transition)		Max. V = 100 fps (Conc. Transition)		Pipe	
Q (ft^3/s)		Q (ft^3/s)		Q (ft^3/s)		Dia	Area
From	Including	From	Including	From	Including	(in)	(ft^2)
0	2.7	0	3.9	0	7.9	12	0.785
2.7	4.3	3.9	6.1	7.9	12.3	15	0.227
4.3	6.2	6.1	8.8	12.3	17.7	18	1.762
6.2	8.4	8.8	12.0	17.7	24.1	21	2.405
8.4	11.0	12.0	15.7	24.1	31.4	24	3.142
11.0	13.9	15.7	19.9	31.4	39.8	27	3.976
13.9	17.2	19.9	24.5	39.8	49.1	30	4.909
17.2	20.8	24.5	29.7	49.1	59.4	33	5.940
20.8	24.7	29.7	35.3	59.4	70.7	36	7.069
24.7	29.0	35.3	41.5	70.7	83.0	39	8.296
29.0	33.7	41.5	48.1	83.0	96.2	42	9.621
33.7	38.7	48.1	55.2			45	11.045
38.7	44.0	55.2	62.8			48	12.566
44.0	49.7	62.8	70.9			51	14.186
49.7	55.7	70.9	79.5			54	15.904
55.7	62.0	79.5	88.6			57	17.721
62.0	68.7	88.6	98.2			60	19.635
68.7	75.8					63	21.648
75.8	83.2					66	23.758
83.2	90.9					69	25.967
90.9	99.0					72	28.274

Source: After Reference *(1)*.

n = assume a conservative roughness coefficient
= 0.013 (see Manning's n – Appendix 9.E)

sf = friction slope of pipe
= $n^2V^2/(2.2R^{4/3})$
= 0.0044 ft/ft

9.B.4.5 Additional Freeboard

1. Additional upstream canal bank freeboard = 0.5 of normal freeboard = (0.5)(1.3) = 0.65 ft, use 0.7 ft. Therefore, the canal bank El. at Sta. A = NWS El. + regular freeboard + additional freeboard = 5407.77 + 1.3 ft + 0.7 ft = 5409.77. Given that the canal profile slope is 0.003 ft/ft, then recommend extending the bank at this elevation a distance of (1.3 + 0.7)/0.003, or say 650 ft, upstream from the siphon to minimize damage that could be caused by overtopping.

2. Additional downstream canal bank freeboard at Sta. M = NWS El. + freeboard = 5406.75 + 1.3 ft = 5408.05. Recommend extending the bank protection at this elevation to the right-of-way or for 50 ft, whichever is less.

9.B.4.6 Trial Transition Invert Elevations

1. The inlet transition invert elevation at the headwall (Sta. C) is based on the hydraulic seal required at the headwall opening and the vertical height of the opening, Ht. The pipe slope affects this vertical dimension because Ht = D/cosα_1 where D is pipe diameter in feet and an estimated α_1 is usually adequate because a small error in α_1 will not significantly affect Ht. The scaled value of α_1 from the Preliminary Siphon Layout is $12^o\underline{+}$:

   Ht = 2.0/cos 12^o = 2.0/0.978 = 2.04 ft

   Hydraulic seal required = 1.5 h_v = 1.5(h_{vp} – h_{v1}) = 1.5 (0.35 – 0.07) = 0.42 ft, which is greater than the 0.25-ft minimum seal required; therefore, 0.42 ft should be used. Transition invert El. C = NWS El. at Sta. A – (1.5 h_v + Ht) = 5407.77 – (0.42 + 2.04) = 5405.31.

   If the transition invert at the cutoff (Sta. B) is set at the canal invert, the difference in invert elevations of the transition, p, is 5406.52 – 5405.31 = 1.21 ft = p. See the foregoing Trial Inlet/Outlet Geometry for p. The allowable maximum p value for the inlet is 3/4(D), and the maximum allowable p value for the outlet is 1/2(D). Therefore, by making the inlet and outlet identical, p cannot exceed 1/2(D), which is 0.5(2.0) or 1.0 ft.

   By using a p value of 1.0 ft, the inlet transition invert El. B will not be the same as the canal but will be El. C + p or 5405.31 + 1.00 ft = 5406.31, which is 0.21 ft lower than the canal invert located at Sta. A. The invert slope for a 10.0-ft long earth transition resulting from the use of p = 1.0 ft should not be steeper than 1V:4H. The actual slope = invert of earth transition to 10.0 = 0.21 to 10.0 = 1 to 48, which is flatter than 1V:4H and therefore permissible.

2. To minimize headwall submergence, set the downstream invert elevation (Sta. G) of the transition at the canal invert. Then, the transition invert El. G = canal invert El. M = 5405.50.

For the inlet and outlet transitions to be identical, p = 1.0 ft. Then, transition invert El. F = El. G – p = 5405.50 – 1.0 = 5404.50.

9.B.4.7 Preliminary Head Loss Analysis

Before establishing the detailed siphon elevations and dimensions, use scaled dimensions and angles as required from the Preliminary Siphon Layout, and determine the approximate total head loss and compare with head provided. This preliminary analysis is to indicate whether the siphon diameter or siphon/canal profile should be revised.

The height of headwall opening (Ht) at station F is:

$$Ht = D/\cos \alpha_2 = 2.0/\cos 12^o = 2.0/0.978 = 2.04 \text{ ft}$$

Submergence of top of opening = $(d_2 + p) - D/\cos \alpha_2 = (1.25 + 1.0) - 2.04 = 0.21$ ft

This submergence should not exceed one-sixth Ht for minimum head loss.

One-sixth Ht = 2.04/6 = 0.34 ft, which is greater than the submergence of 0.21 ft. Therefore, the loss for the outlet transition is minimum and may be calculated using the equation $0.7\Delta h_v$ (otherwise, $1.0\Delta h_v$ should be used).

From the Preliminary Siphon Layout, the drop in water surface elevation (available head) = NWS El. Sta. A – NWS El. Sta. M = 5407.77 – 5406.75 = 1.02 ft.

Total siphon head loss with 10% safety factor = 1.1 (inlet transition convergence loss + pipe friction loss + bend losses + outlet transition divergence loss).

Pipe length scaled = 72 ft

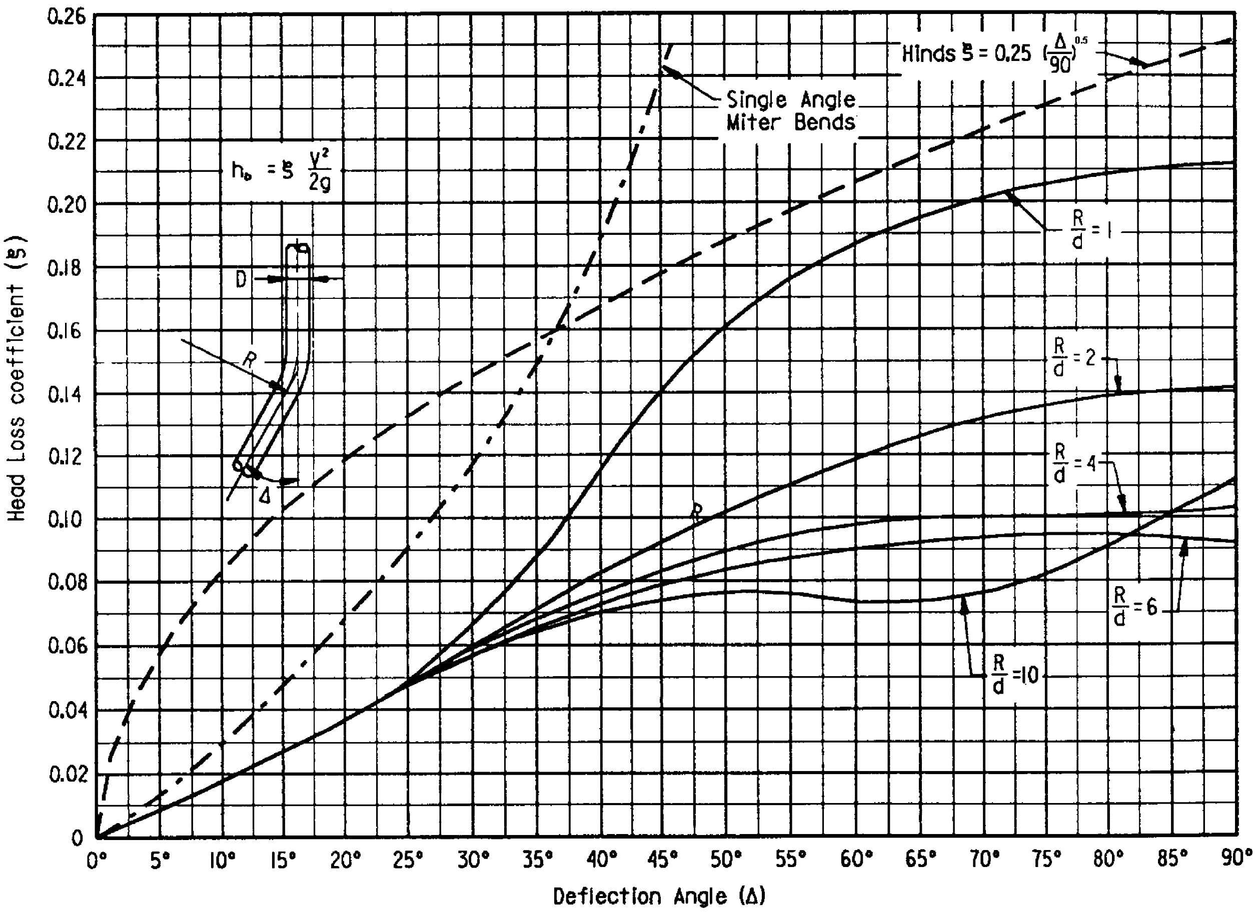
Pipe bend angles scaled = approximately 12^o (assume single angle mitered bends). From the following figure, the bend losses as a function of the velocity head is approximately 0.04.

Approximate total head loss $H_L = 1.1 (h_i + h_f + h_b + h_o)$ where h_i is inlet loss, h_f is siphon friction loss, h_b is siphon bend loss and h_o is outlet loss:

$$H_L = 1.1[0.4\Delta h_v + (\text{siphon length})(s_f) + (h_{vp})(2) + 0.7\Delta h_v]$$
$$H_L = 1.1[0.4(0.35 - 0.07) + (72)(0.0044) + ((0.04)(0.35))2 + 0.7(0.35 - 0.07)]$$
$$= 1.1[0.11 + 0.32 + 0.03 + 0.20] = 1.1(0.66) = 0.73 \text{ ft}$$

Head provided by canal profile = 1.02 ft, which is 0.29 ft more than the 0.73 ft required for this preliminary layout. At this point, one is reasonably certain that one can enter the foregoing transition, canal and freeboard elevations on the Final Siphon Layout found later in this Example.

Note that the foregoing excess head will cause a slight drawdown in the canal upstream from the siphon and will thus result in faster than normal velocities for a short distance. For now, assume that this velocity is noneroding so it is not necessary that the profile for the canal or the size of siphon be revised. See Figure 9.B-4.



Source: Reference *(1)*, p. 360.

FIGURE 9.B-4 — Head Loss

9.B.4.8 <u>Outlet Dimensions</u>

(Note: See Figure 9.B-3 for trial Inlet/Outlet Geometry)

1. TRANSITION DIMENSION, y. Dimension y should be determined so that the freeboard provided at the cutoff will be at least 0.5 ft:

   $y = (\text{NWS El. Sta. A} - \text{El. B}) + F_b = (5407.77 - 5406.31) + 0.5 = 1.46 + 0.5 = 1.96$ ft

2. TRANSITION DIMENSION, a. Freeboard at the transition headwall for siphon diameters 24 in and smaller may be the same as the freeboard at the cutoff. Therefore, the top of the headwall is set at the same elevation as the top of the wall at the cutoff and is equal to:

   El. top of wall = El. B + y = 5406.31 + 2.0 = 5408.31
   a = El. top of wall – El. C
   = 5408.31 – 5405.31 = 3.0 ft

3. TRANSITION DIMENSION, C. Initially, try the standard transition length set forth on the Trial Inlet/Outlet Geometry Figure of C = D + 3.0 ft:

   C = 2.0 + 3.0 = 5.0 ft

For this Example, assume the bottom width of the upstream canal was measured to be 7.0 ft, which suggests some energy loss for flows to enter a transition having a 5.0 ft inlet width. It is necessary to ensure that this loss will not increase the upstream flow depth to where the freeboard is exceeded. Because the transition wingwalls are at 45°, assume a loss equal to 0.5 h_v, and that as a worse case critical velocity occurs at the transition inlet. The upstream canal velocity V_1 was previously determined as 2.1 ft/s critical velocity in a rectangular section, for the 5.0-ft transition width can be determined from $d_c = 0.315\ [(Q/B)^2]^{1/3}$ and $V_c = (Q/(Cd_c)$:

$$d_c = 0.315\ [(15/5)^2]^{1/3} = 0.66 \text{ ft}$$
$$V_c = 15/((0.66)(5)) = 4.5 \text{ ft/s}$$

Therefore, the maximum potential depth increase (BW) would be:

$$BW = 0.5\ (4.5^2/2g - 2.1^2/2g) = 0.12 \text{ ft.}$$

The maximum potential backwater is substantially less than the 0.7 ft of additional freeboard provided above, so the C = 5.0 ft is considered satisfactory.

4. DEPTH OF TRANSITION CUTOFF. The Trial Inlet/Outlet Geometry figure reflects a transition cutoff depth of 3.0 ft. At this point, there appears to be no reason to change this standard as velocities (even critical velocity at the transition inlet) are low as are the canal depths.

5. CONCRETE TRANSITION LENGTH, L. The transition length is a function of the siphon diameter with a lower limit of 1:4:

   $$L = (3)(\text{pipe dia.}) = (3)(2.0) = 6.0 \text{ ft}$$

   Also, 6:(5.0 – 2.0)/2 is 6:1.5 or 1:4, so this criteria is satisfied.

6. PIPE EMBEDMENT. Embedment details for the siphon at the headwalls and construction requirements of the siphon bends are a structural issue beyond the scope of this *Manual*; see Reference *(1)*, Section 9.B.5.

It is now reasonably safe to transfer the foregoing dimensions to the Final Siphon Layout pending only the Final Siphon Head Loss Check and the Blowback Check.

9.B.4.9 Thrust At Bends

Because the hydraulic thrust caused by the bend is directed into the siphon foundation, stability for the bend is probably sufficient. Unusually poor foundation conditions in addition to high heads, large-diameter siphon and large deflection angles may, however, require that a thrust block be considered after determining the reaction where poor foundation materials are encountered. For this Example, consider the foundation material as being adequate to resist the hydraulic thrust.

9.B.4.10 Final Siphon Layout

Using the elevations, dimensions and earth slopes previously computed or given, determine the final structure stationing, pipe elevations and siphon slopes, and complete the Final Siphon Layout at the end of this Subsection.

Stations C and F at the transition headwalls are controlled by the roadway earthwork dimensions and side slopes and, from the dimensions already entered on the Final Siphon Layout, it can be determined that Station C must be at least 34.36 ft upstream from the roadway centerline:

Sta. C = Sta. J – 34.36 ft
= Sta. (150+00) – 34.36 ft
= Sta. 149+65.64 (or less). Use Sta. C = Sta. 149+65

Sta. B is then:

Sta. B = Sta. C – 6.0 ft
= Sta. (149+65) – 6.0 ft = Sta. 149+59

and Sta. A becomes:

Sta. A = Sta. B – 10.0 ft
= Sta. (149+59) – 10.0 ft = Sta. 149+49

The small difference in the given value of Station A (149+47) and the computed Station (149+49) is not significant enough to require any canal invert profile changes. Stations F, G and M are determined in the same manner as Stations A, B and C. Again, from the Final Siphon Layout, it can be determined that Station F must be at least 30.38 ft from the roadway centerline:

Sta. F = Sta. J + 30.38 ft
= Sta. (150+00) + 30.38 ft
= Sta. 150+30.38 (or greater), Use Sta. 150+31

Sta. G is then:

Sta. G = Sta. F + 6.0 ft
= Sta. (150+31) + 6.0 ft = Sta. 150+37

Sta. M becomes:

Sta. M = Sta. G + 10.0 ft
= Sta. (150+37) + 10.0 ft = Sta. 150+47

Here again, the difference between the given and computed values for Station M is small and will not require any canal invert profile changes.

Stations D and E are selected to ensure that a 2.0-ft minimum of earth cover on the siphon is provided at the roadway ditches. The inverts of the V-ditches are located 15.25 ft from the roadway centerline. Therefore, the siphon bend inverts should be located about 16 ft from each side of the centerline of the roadway:

Sta. D = Sta. J – 16.0 ft
= Sta. (150+10) – 16.0 ft = Sta. 149+84

El. D is determined by subtracting the siphon diameter, the shell thickness and the minimum cover from the elevation of the ditch invert:

El. D = (5407.00 – 1.5 ft) – (2.0 ft + 0.25 ft + 2.0 ft) = 5401.25

Similarly, Sta. E = Sta. J + 16.0 ft =
Sta. (150+00) + 16.0 ft = Sta. 150+16

El. E is determined by subtracting the product of the distance between Stations D and E and the siphon slope 0.005 (which is a minimum slope) from El. D:

El. E = El. D – (32 ft)(0.005) = 5401.25 – 0.16 ft = 5401.09

Upstream siphon slope (S_1). Slope of the siphon between Stations C and D is calculated as follows:

Horizontal distance = Sta. D – Sta. C = (Sta. 149+84) – (Sta. 149+65) = 19 ft
Vertical distance = El. C – El. D = 5405.31 – 5401.25 = 4.06 ft

S_1 = (vertical distance/horizontal distance) = 4.06 ft/19 ft = 0.214

Angle of the slope is the angle whose tangent is 0.214, $\alpha_1 = 12^o05'$

Downstream siphon slope (S_3). Determine slope of the siphon between Stations E and F:

Horizontal distance = Sta. F – Sta. E = Sta. (150+31) – Sta. (150+16) = 15 ft

Vertical distance = El. F – El. E = 5404.50 – 5401.09 = 3.41 ft

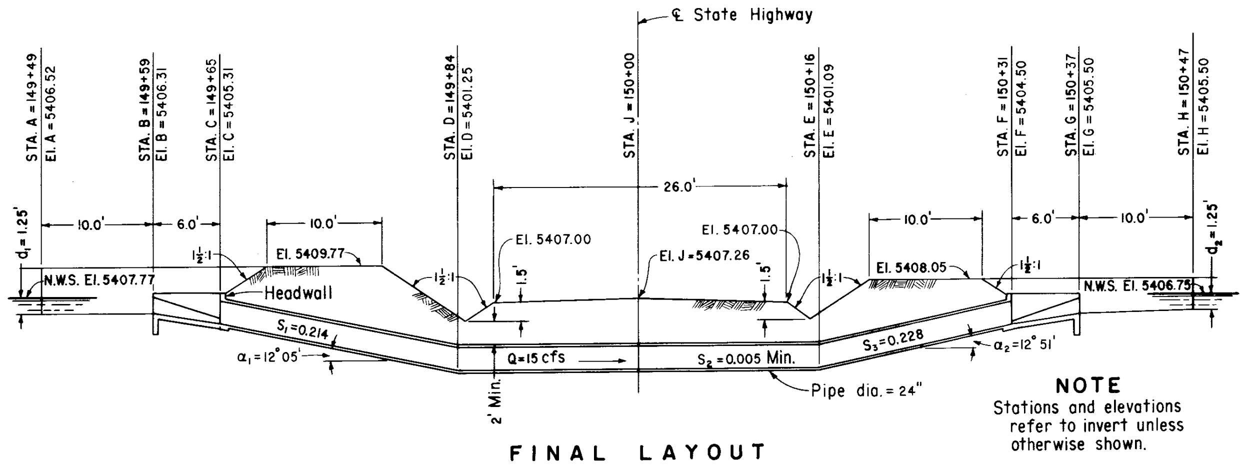
S_3 = (vertical distance/horizontal distance) = 3.41 ft/15 ft = 0.227

Angle of the slope is the angle whose tangent is 0.227, $\alpha_2 = 12^o48'$

The foregoing expected final dimensions and angles can be entered on the following Final Siphon Layout (Figure 9.B-5).

9.B.4.11 Final Head Loss Check

It is now necessary to ensure that the dimensions, angles and elevations shown on the Final Siphon Layout do not materially change the final head loss from that determined in the Preliminary Head Loss Analysis.



Source: After Reference *(1)*.

FIGURE 9.B-5 — Final Layout of Inverted Siphon

Total final siphon head loss with 10% safety factor = 1.1 (inlet transition convergence loss + siphon friction loss + siphon bend losses + outlet transition divergence loss), or:

$$H_L = 1.1\,(h_i + h_f + h_b + h_o)$$
$$H_L = 1.1[0.4\Delta h_v + (\text{siphon length})(s_S) + (h_{vp})(2 + 0.7\Delta h_v)]$$

Determine siphon length:

From Station C to Station D:

$$\text{Length} = (\text{Sta. D} - \text{Sta. C})/\cos\alpha_1 = 19\text{ ft}/0.978 = 19.4\text{ ft}$$

From Station D to Station E: Because the siphon slope is relatively flat, use horizontal distance equals:

Sta. E – Sta. D = (150+16) – (149+84) = 32 ft

From Station E to Station F:

$$\text{Length} = (\text{Sta. F} - \text{Sta. E})/\cos\alpha_2 = 15\text{ ft}/0.975 = 15.4\text{ ft}$$

Total siphon length = 19.4 + 32.0 + 15.4 = 66.8 ft

Therefore, the total head loss in the siphon is:

$$H_L = 1.1[0.4(0.35 - 0.07) + (66.8)(0.0044) + (0.04)(0.35)(2) + 0.7(0.35 - 0.07)]$$
$$= 1.1(0.11 + 0.29 + 0.03 + 0.20) = 0.69\text{ ft}$$

Because the head provided by the canal profile (1.02 ft) is greater than the head required (0.69 ft), a slight water surface drawdown will definitely occur for a short distance upstream from the siphon. Although this excess head will cause faster than normal velocities, it is unlikely they will reach or exceed the inlet's critical velocity of 4.5 ft/s estimated when determining the Inlet/Outlet Dimensions. For this design Example, assume that these velocities are still noneroding, so neither the canal profile nor the siphon size need be revised. Unless the Blowback Check shows otherwise, the Final Siphon Layout dimensions and angles can be considered as final.

9.B.4.12 Erosion Protection

1. OUTLET. The width of the end of the outlet transition is 5.0 ft whereas, for the purpose of this Example, we previously assumed the canal's bottom width was measured to be 7.0 ft. This subjectively indicates a higher average outlet velocity than the 2.1 ft/s occurring naturally in the canal. More specifically, the velocity at the end of the outlet transition would be approximately $Q/(Cd_2)$:

   $V = 15/((1.25)(5)) = 2.4 \text{ ft/s} > 2.1 \text{ ft/s}$

   This is not considered to be a significant increase in light of the 3.0-ft cutoff wall, so outlet protection is not recommended. A tractive shear analysis as provided below for the inlet could be used to verify this finding.

(2) INLET. It was previously determined that, should critical velocity somehow occur at the entrance to the inlet, the velocity could reach 4.5 ft/s. It is unlikely that this velocity will materialize and, if it does, will cause erosion. However, as a safety measure, line the upstream earth canal transition with a well-graded rock of a size to resist the expected tractive shear, τ, where d_c and S_c are the critical upstream depth and hydraulic gradient at the entrance to the inlet and ω is the unit weight of water:

d_c = 0.66 ft
V_c = 4.5 ft/s
n = say 0.04 for small stone (probably conservative)
A_c = $(0.66)(5) = 3.3 \text{ ft}^2$
WP_c = $2(0.66) + 5 = 6.3 \text{ ft}^2$
R_c = $A_c/WP_c = 3.3/6.3 = 0.52$ ft
$R_c^{2/3}$ = $0.52^{2/3} = 0.65$
S_c = $[V_c\, n/R_c^{2/3}]^2 = [4.5\,(0.04)/(1.486)(0.65)]^2 = 0.035$ ft/ft
τ = $\gamma d_c S_c = (62.4)(0.66)(0.035) = 1.44 \text{ lbs/ft}^{2-}$

If the critical tractive shear for the natural channel material was less than 1.44 lbs/ft^2 then a stone size would be selected using the practices set forth in the Channel Chapter of this *Manual*. This stone would be placed on the upstream reach transition bed and banks for a distance of three times the critical depth or (3)(0.66) = say 2 ft.

9.B.4.13 Siphon Collars

Assume that collars are not needed to discourage burrowing animals, but that collars may be necessary to slow the percolation of water along the siphon. The difference in elevation between the canal water surface and the roadway ditch is 2.3 ft (ΔH). Assume that the weighted creep ratio, percolation factor, required to prevent piping in the material encountered at the site was found to be 3.0. Determine the weighted creep length (L_W) from the inlet transition to the first roadway ditch assuming that the seepage water flows along the bottom side of the siphon from Station B to Station D; then along the outside of the siphon to the top of the siphon; and finally through the earth to the ditch invert. Weighted creep lengths are derived by multiplying the path length by one if the path is vertical and between structure and earth; by one-third if the path is horizontal between structure and earth; and by two if the path is through earth:

L_w = (2)(vertical dimension of cutoff)(1) + (Sta. D – Sta. B)(1/3) + (outside diameter of siphon) (1) + (earth cover on siphon) (2)

= (2)(2.0)(1) + (25)(0.33) + (2.5)(1) + (2)(2)

= 4.0 + 8.3 + 2.5 + 4.0 = 18.8 ft

Determine the percolation factor (PF) that this weighted creep distance will provide:

$PF = L_w/\Delta H = 18.8/2.3 = 8.2$

Because the percolation factor provided (8.2) is greater than that assumed to be necessary (3.0), siphon collars are not needed.

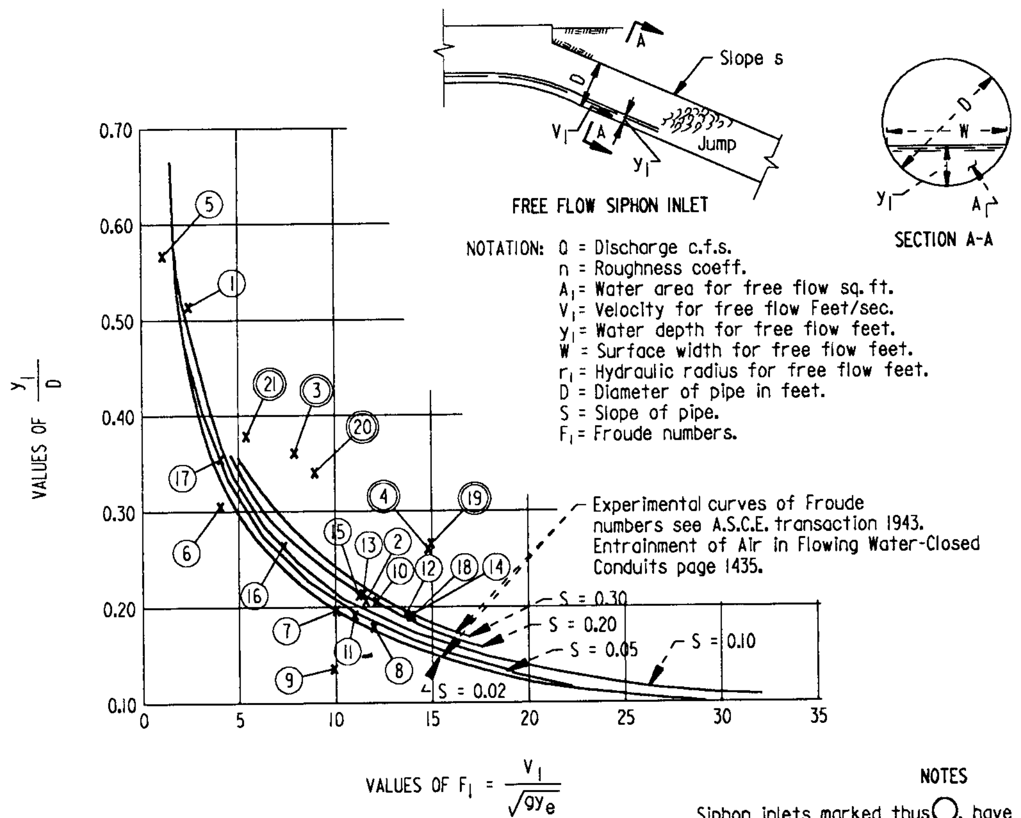
9.B.4.14 Blowoff Requirement

The structure is by definition considered to be short, and the siphon could be drained by pumping from the ends so that a blowoff system would not be required. However, to reduce long-term maintenance costs and the need to mobilize a pump, a blowoff will be provided. Use a 8-in minimum gate valve in a pit to expedite draining and to facilitate the flushing of trapped sediment. An inspection manhole will not be required due to this being a short, small siphon.

9.B.4.15 Blowoff Check

This siphon structure is defined by the criteria as short and not likely to have blowback because the air that might be entrained due to a possible free-flow inlet and hydraulic jump will probably be carried downstream and exhausted at the downstream end of the siphon. The short length also precludes trapping air entrained from other sources. However, a routine check is advisable — partly to illustrate the procedure.

Assume the canal company elects to only pass about half the canal capacity of 15 ft^3/s or 7 ft^3/s. Compute the following hydraulic values as required to use the blowback criteria graph in Figure 9.B-6. From standard hydraulic tables commonly used to determine uniform flow at partial flow depths in circular sections, we can compute:



	LOCATION	TYPE OF PIPE	Q	D	n	s	$\frac{y_1}{D}$	F_1
1	Yakima River - Yakima Project	Conc.	925	111"	.010	.82	.511	2.47
2	Malheur River - Owyhee Project	Steel	325	80"	.010	.213	.205	11.60
3	Basin Siphon - King Hill	Wood	250	57"	.012	.17	.360	7.90
4	Basin Siphon - King Hill	Conc.	250	54"	.012	.61	.260	14.90
5	San Diego Sta. 620+00 N	Conc.	95	54"	.010	.0033	.567	1.25
6	San Diego Sta. 961+00 N	Conc.	95	54"	.010	.029	.305	4.10
7	San Diego Sta. 1146+00 N	Conc.	95	54"	.010	.19	.194	10.00
8	San Diego Sta. 1545+15 N	Conc.	95	54"	.010	.2716	.177	11.90
9	San Diego Sta. 1817+50 N	Conc.	95	72"	.010	.1935	.134	9.95
10	San Diego Sta. 1873+50 N	Conc.	95	48"	.010	.3053	.202	12.15
11	San Diego Sta. 2200+02 N	Conc.	95	54"	.010	.2198	.185	10.95
12	San Diego Sta. 1303+56 S	Conc.	95	48"	.010	.3643	.192	13.70
13	San Diego Sta. 1217+50 S	Conc.	95	48"	.010	.2563	.212	11.35
14	San Diego Sta. 1073+00 S	Conc.	95	48"	.010	.3714	.19	13.83
15	San Diego Sat. 1073+00S	Conc.	95	48"	.012	.3714	.211	11.20
16	San Diego Sta. 419+00 S	Conc.	95	48"	.010	.104	.262	7.33
17	San Diego Sta. 171+96 S	Conc.	95	48"	.010	.0336	.355	4.17
18	San Diego Sta. 79+92 S	Conc.	95	48"	.010	.377	.190	13.80
19	High Mesa-Uncompaghre	Steel	42	26"	.010	.535	.264	15.00
20	Lake Valley Crossing P.G.E.	Steel	35	24"	0.13	.367	.349	8.95
21	Lake Valley Crossing P.G.E.	Steel	25	24"	0.16	.208	.378	5.43

Source: U.S. Bureau of Reclamation, "Design of Small Irrigation Structures"

NOTES

Siphon inlets marked thus ◯, have given trouble in operation and air inlets were installed in some casses to relieve the blowing back of air and water.

All other siphons have not given trouble in operation.

Study made indicates that free flow siphon inlets designed so that Froude number will not fall above the critical curves established by experiments will give satisfactory performance.

Procedure to determine Froude number.

a. Calculate V_1 with Manning's Formula.

b. Calculate y_1

c. Calculate $W = 2\sqrt{(D-y_1)y_1}$

d. Calculate $y_e = \frac{A_1}{W}$

e. Calculate Froude number $F_1 = \frac{V_1}{\sqrt{gy_e}}$

FIGURE 9.B-6 — Design of Free Siphon Inlets (Blowback Criteria Graph) ***(1)***

$$(Qn)/D^{2.67}S^{1/2}) = ((7)(0.013))/((2.0^{2.67})(0.21^{0.5})) = 0.03$$
$$Y_1/D = 0.17 \pm \text{ or } Y_1 = (0.17)(2.0) = 0.34 \text{ ft}$$
$$A_1/D^2 = 0.089 \pm \text{ or } A_1 = (0.089)(2.0^2) = 0.36 \text{ ft}^2$$
$$V_1 = Q/A = 7/0.36 = 20 \pm \text{ ft/s}$$
$$W_1 = 2(DY_1 - Y_1^2)^{0.5} = 2((2.0)(0.34 - 0.12))^{0.5} = 1.33 \text{ ft, use } 1.5 \text{ ft}$$
$$Y_e = A_1/W_1 = 0.36/1.5 = 0.24$$
$$F_1 = V_1/(gY_e)^{0.5} = 20/((32.2)(0.24))^{0.5} = 7.2$$

By plotting F_1, and Y_1/D on the blowback criteria graph (Figure 9.B-6), we can see that, at a partial flow of 7 ft^3/s, the siphon will not induce blowback. Except for the following Safety Features, the Final Siphon Layout dimensions and angles are completed and may be used for final plans.

9.B.4.16 <u>Safety Features</u>

Provide a removable grate at the inlet and outlet. Assuming canal debris blocks the inlet, it will be necessary for the grate to structurally sustain a vertical load equal to the weight of water that could be trapped above it.

Because there is only approximately a 3-ft difference between the roadway ditch bottom and the top of the canal bank, a serious erosion problem is not expected should the canal be overtopped. If a serious erosion problem had been expected, then an overflow weir would be provided (either separate or monolithically with the inlet transition) that discharged into a receiving ditch or chute wasteway to convey excess flows to either such features as the nearest drain or to some temporary storage facility.

9.B.5 REFERENCES

(1) US Bureau of Reclamation, *Design of Small Canal Structures*, 1978.